Biaxial testing to investigate soil-pipe interaction of buried fiber reinforced cement pipe

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Abstract: Three tests have been performed to develop baseline information on the behavior of fiber reinforced cement (FRC) pipe under biaxial loading. Pipes were instrumented with strain gauges to measure circumferential strains at various locations around the circumference, and with linear potentiometers to capture changes in vertical and horizontal pipe diameter. The first two tests examined the response of wet and dry pipes under conditions simulating steadily increasing overburden pressure in an embankment loading condition, reaching earth pressures equivalent to more than 15m. The third test examined the time-dependent response of a second, wet pipe under sustained pressure of 180kPa. All tests were performed in a loose granular backfill with relatively low soil modulus.

Elastic soil-pipe interaction theory shows that increases in pipe deformation lead to redistribution of loads to the surrounding ground and reductions in bending moments. Preliminary calculations for the three pipe tests indicate that even in the low modulus backfill, the pipe deformations were sufficient to reduce applied earth pressures on the pipes, and the bending moments within them. These pipes exhibit 'semi-rigid' behavior, where bending moments are reduced below levels experienced by a 'rigid' pipe. Those bending moment decreases were greater for the wet pipe than for the dry pipe, since the wet specimens had lower manufactured wall thickness and wet FRC has lower modulus. Reductions in the modulus of the fiber reinforced cement material were estimated to decrease moments by about 30% by the end of the 50 hour time period of the third test, and by approximately 36% at the peak overburden pressure (300kPa) reached during the second test.

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Key Words: Fiber reinforced cement pipe, bending moments, curvature, extreme fiber strains, semi-rigid pipe behavior, time dependent response.

INTRODUCTION

Fiber reinforced cement pipes have been recently introduced into the North American pipe market, and work has commenced to investigate their soil-pipe interaction when used as culvert structures. The pipes are manufactured by James Hardie using cellulose fibers to add tensile strength and ductility to a cement-silica matrix, and are based on pipes produced in Australia to AS4139 (1993).

Three fiber reinforced cement (FRC) samples manufactured in Florida were buried in the biaxial test cell at Queen's University, and were tested under simulated embankment loading. Aspects of the soil-pipe interaction being investigated include the differences between dry FRC and wet FRC pipe (with the latter saturated in accordance with AS4139). Also investigated is the time dependent response of the FRC material, and the progressive transfer over time of non-uniform earth loads from the pipe to the soil surrounding it. Measurements of pipe deformation and surface strain are used here to characterize the pipe response. Calculations using elastic soil-pipe interaction theory provide preliminary information about the semi-rigid response of these pipe structures, and the level of moment reductions that occur as a result of pipe deformations and load transfers from the pipe to the surrounding ground. More detailed soil-pipe interaction analysis of the biaxial buried pipe tests will be undertaken subsequently using nonlinear finite element analysis, to calibrate those analyses.

PIPE SPECIMENS AND TEST INSTRUMENTATION

Pipe Specimens

Each of the three pipes had 381-mm (15-inch) internal diameter. The pipes were cut to a length of 1950mm, so that the ends of the pipe would not touch the sidewalls of the 2m long test cell. This means that the ends of the pipe are not restrained, but are free to undergo axial (longitudinal) expansion just as they would if placed with bell and spigot connections in the field (with spigot at approximately the middle position axially within the bell).

The pipe specimens for the first, second and third tests had wall thicknesses of 30 mm, 25.4mm and 25.4mm, respectively. The two wet pipe specimens were soaked in a water bath for 29 and 34 days, respectively, in accordance with the testing procedures set out in AS 4139.

Pipe instrumentation

Figures 1 and 2 show a typical pipe cross-section and the location of both the linear potentiometers and the resistance strain gauges. Instrumentation was placed at two sections denoted A and B, each 500mm on either side of the pipe centerline.

Linear potentiometers oriented vertically and horizontally were placed within the pipe to measure the changes in vertical and horizontal pipe diameter, ΔD_v and ΔD_h respectively. Biaxial strain gauges were used to measure the circumferential and the axial strains at five separate locations around the pipe circumference: at the Crown, Invert and both Springlines, as well as in the lower right hand Haunch of the pipe, though only circumferential strains are reported here. By duplicating the instrumentation at the second section, the reproducibility of some of the test results was established, and the second set of measurements ensured that data was collected at all locations shown in Fig.1 where some of the gauges at the first section were inoperative (particularly important for the wet pipe tests). Resistance strain gauges were placed by first preparing the pipe surface with a thin layer of adhesive from MicroMeasurements group. The gauges were then fixed to the surface with that same adhesive. Prior to soaking, the strain gauges for these two ‘wet pipe’ samples were also covered with a layer of melted wax as an additional sealant.

Test Cell Arrangement

The biaxial test cell at Queen’s University is a high strength steel box with dimensions 2m by 2m in plan and 1.6m in height. The arrangement of the pipe and instrumentation for these tests are shown in Fig. 3. Each of the pipes was placed horizontally equidistant from the sidewalls at a height of 600mm above the base of the cell, leaving backfill of 810-mm width on both sides of the pipe (more than two pipe diameters). Design of the cell (Brachman et al., 2000) included a special lubricated, multi-layer treatment (Tognon et al., 1999) to minimize the sidewall friction. Finite element analysis to study the influence of the side boundaries has established that at this distance, the influence of the lubricated sidewalls is small (less than 5%) under the action of uniform overburden stresses simulated at the top surface, Dhar (2002). Both finite element calculations and experimental measurements of earth pressure confirm that at least 95% of the applied overburden pressure reaches the level of the pipe.

Poorly graded sand (uniformity coefficient, C_u = 1.46; coefficient of curvature, C_c = 0.94) was used as the backfill material, as described by Lapos and Moore (2002). The backfill soil was placed loosely in the cell, at a density of about 1300-kg/m^3, which is approximately 85% of the maximum standard Proctor density. Earth pressure cells were used to measure the vertical and horizontal stresses at the springline and at 200mm below the top surface of the soil. Two settlement plates were positioned adjacent to the pipe, to monitor the vertical soil movements. Figure 4 shows the dry pipe specimen during the burial process. A working platform was slung within the cell above the level of the backfill, to ensure that backfill remained undisturbed prior to testing (the platform was removed to record the image shown in Figure 4).

An air bladder was used to apply uniform pressures on top of the soil, Figure 3. Tests were conducted in pressure increments of 20kPa, with each increment allowed to remain for 8 minutes. Since the uniform poorly graded backfill acts as an efficient vapor barrier, the wet pipe samples were not expected to dry out during the 2 to 5 day period over which the pipe burial and testing was conducted. The first wet pipe sample was visually inspected when it was exhumed, this being a total of 4 days after it taken from the water bath and installed in the cell. This inspection indicated that little surface drying of the pipe sample had occurred.

Pipe Maximum Overburden Pressures

The three tests featured the following pipe specimens and loading histories:

a. Test 1 featured the pipe specimen with 30mm wall thickness, tested in a dry state to a maximum overburden pressure of 240kPa (equivalent to a burial depth of about 18m in this particular backfill material)

b. Test 2 featured a pipe specimen with 25mm wall thickness, tested in a wet state to a maximum overburden pressure of 300kPa (equivalent to a burial depth of about 23m in this backfill)

c. Test 3 featured a pipe specimen with 25mm wall thickness, tested in a wet state to a maximum overburden pressure of 180kPa (equivalent to a burial depth of about 13m), and held at that load level for two days

Each of the pipes was inspected after exhumation and found to have retained its structural integrity, even though the maximum burial loads exceeded the current values of allowable and ultimate burial depth.

GROUND RESPONSE

Figures 5 and 6 show different aspects of the soil response adjacent to the pipe where stress cells and settlement plates were installed to monitor the vertical soil response. Figure 5 shows the vertical response of the settlement plates during Tests 1 to 3, these indicating that the loosely placed backfill had reasonably consistent stiffness. Figure 6 provides the constrained soil modulus M_S, calculated from the settlements S_v shown in Figure 5, the applied vertical stresses σ_v, and the height of the soil column h = 500 mm that is compressing below the settlement plate:

$$M_s = \frac{\sigma_v h}{S_v} \tag{1}$$

Again, the constrained soil modulus is reasonably consistent. It commences at approximately 2.5MPa, and increases with increasing overburden pressure to reach between 6 and 7.5MPa at the highest pressure levels tested. Equivalent Young's modulus for this backfill is about 35% lower (based on a value of Poisson's ratio for this loose material of approximately 0.35), making the modulus of this test soil lower than (conservative relative to) the granular and silty backfill materials recommended for use in design by McGrath (1998) and Moore (2001) (based on constitutive data obtained by Selig, 1988).

PIPE DEFORMATIONS

Figure 7 provides details of the changes in vertical and horizontal pipe diameter with increasing overburden pressures. These results indicate that:

- Deformations are small, representing less than one percent of pipe diameter at overburden pressures of 180kPa (equivalent to 13m or 40ft of burial in the test soil)

- Vertical diameter change (decrease) ΔD_V is essentially equal and opposite to the horizontal diameter change (increase) ΔD_H
- None of the pipes reached the limits of ductility or structural capacity during these tests
- Deformations for the dry pipe specimen in Test 1 are about half those for the wet pipe specimens at any specific level of overburden pressure (or equivalent burial depth)
- Diameter changes are essentially linear with increasing overburden pressure, up to about 1.5mm in the wet pipe samples and to at least 2.0mm in the dry sample; beyond that point, the stiffness of the wet pipes begins to decrease (leading to additional levels of load transfer to the surrounding soil, as discussed in a subsequent section).

Elastic soil-pipe interaction solutions like those of Burns and Richard (1964) or Hoeg (1968) clearly indicate that FRC pipes have hoop stiffnesses that are large compared to the surrounding soil, so that hoop strains are small and the classic 'ovaling' response results, with $\Delta D_H = - \Delta D_V$. For a stiff pipe of this type, the changes in diameter are largely controlled by the flexural rigidity of the pipe wall, EI. Taking into consideration the higher manufactured wall thickness of the specimen used in Test 1, the dry pipe modulus is about 15% higher than that of the wet pipe. The role of pipe modulus on the level of load transfer from the pipe to the surrounding soil is discussed in detail later in the paper.

Fiber reinforced cement exhibits time dependent (effectively viscoelastic) behavior. To investigate this phenomenon, the third test on wet pipe concluded with a 48.5 hour period of sustained overburden pressure of 180kPa, between time periods 1.5 hours and 50 hours. Pipe deformations during that time are shown in Figure 8, plotted against the log of time. These reveal the effect of ongoing FRC pipe deformation with time. There is an approximately 12%

increase in pipe deformation over that period of about 1.6 cycles of log time. The change in pipe diameter with log of time is approximately linear during that period, though the rate of deformation with log of time appears to be decreasing towards the end of that period. This decrease may be because of the growing importance of ground resistance to pipe deformation. The influence of that ground support on control of pipe wall strains and therefore bending moments is investigated in subsequent sections.

EXTREME FIBER STRAINS

Each of the pipe specimens was instrumented with resistance strain gauges to measure circumferential strains at the extreme fibers. Table 1 shows a complete set of readings from each of the pipe tests at the maximum level of overburden pressure.

For the dry pipe at a maximum overburden pressure of 240kPa, all but two of the gauges operated successfully. Largely consistent values of tensile strain were obtained on the inner surface of the Crown and Invert, and at the outer surface of the Springlines. A somewhat lower value of tensile strain was obtained on the outside surface of the right hand Springline at section A, and while this value may be correct, it was eliminated from subsequent calculations to provide conservative estimates of change in pipe curvature. Compressive strain values on the outer surface at Crown and Invert, and the inner surface at the Springlines were also largely consistent. In each case they are higher in magnitude than the tensile strains on the other side of the pipe wall, as a result of the compressive wall thrusts induced by the external earth pressures. Gauge failure meant that only two of these compressive strain measures were obtained, and they exhibited a larger degree of variation than those at Springlines.

Strain gauge measurements were considerably more difficult to obtain for the wet pipe specimens. The integrity of many of the gauges, and/or their circuits, attached to the specimen used during Tests 2 and 3 appeared to have been compromised during pipe saturation. For example, test pipe 3 had just five successful readings out of sixteen, with the others giving no reading at all, or abandoned due to very low values. The inner surface gauge at the Crown for test 2 (section B) gave readings that were approximately double those at other comparable locations, and this reading was not used in subsequent calculations since it would otherwise imply the presence of a tensile hoop force at that location that is not possible. Readings from the Invert strain gauge on the inner surface of section B, Test 3, were abandoned because the strain gauge record over the entirety of the test exhibited a number of discontinuities (large jumps) in consecutive readings.

All haunch gauge readings were substantially lower than those at the Crown, Invert and Springlines, since bending moments change sign between the Invert and Springlines and are approximately zero at the haunch positions. Dhar and Moore (2002) analyzed the effect of poor soil support under the pipe haunches, and showed that considerations of local bending due to poor soil support under the haunches of profiled thermoplastic pipes are not needed for stiffer structures like FRC pipe.

Also included in Table 1 are the set of strain gauge readings at the end of the period of sustained load applied in Test 3. These reveal that the successful readings after two hours are still largely successful, with just one of the five readings at the earlier time period becoming unreliable (the inner surface strain on the left Springline at instrumented section A). Generally the level of strain has increased by about 8%.

HOOP STRAIN AND CURVATURE

The strains at the inner and outer fibers can be used to assess the level of hoop strain and change in pipe curvature at those locations. The averaged values of extreme fiber strains shown in Table 2 were used to calculate hoop strain (the average of the inner and outer surface strains) and the change in pipe curvature (the difference in the inner and outer surface strains divided by the distance between these two positions).

Test pipes 1 and 2 exhibited similar values of tensile strain at the inner surface of the Crown and Invert and the outer surface of the Springlines. Therefore, the missing value of tensile strain for Test pipe 3 (at the Crown and Invert) was assumed equal to the Springline value (likely reasonable, though the compressive strain values for Test pipe 3 are seen to be higher at the Springline).

In each case, the change in curvature at crown and invert is opposite in sign and within 10% of the magnitude of the Springline values. This is consistent with the theoretical understanding of a pipe placed in uniform soil, Moore (2000). Haunch values of curvature change are one order of magnitude lower, as expected, since elastic soil-pipe interaction theory would suggest moment at those locations are close to zero.

Hoop strains are higher at the Springline than the Crown and Invert, again in accordance with a conventional understanding of pipe response. Hoop strains at the haunches are less consistent, being lower than the Crown and Invert values even though elastic soil-pipe interaction theory would suggest that the hoop strains at the haunches lie mid-way between the Springline and Crown/Invert values.

ASSESSMENT OF SEMI-RIGID PIPE RESPONSE

One of the objectives of this experimental study is to quantify the level of ground support that these FRC pipes experience, and the extent to which bending moments are reduced as a result of semi-rigid soil-pipe interaction. Subsequent work will be performed using elastic theory and non-linear finite element analysis to study the soil-pipe interaction in more detail. For the moment, the measurements of diameter change will be used to estimate the extent to which soil-pipe interaction theory benefits these structures.

A rigid pipe under geostatic earth load has the decrease in vertical pipe diameter controlled by the flexural rigidity of the pipe $E_{pipe}I_{pipe}$, Moore (2000), since the pipe stiffness greatly exceeds the soil stiffness. However, a buried flexible pipe has changes in pipe diameter controlled by the soil modulus, Moore (2000), since pipe stiffness $E_{pipe}I_{pipe}$ is negligible compared to the soil. Expressed as a function of the constrained modulus of the soil = M_S, coefficient of lateral earth pressure = K, overburden stress = σ_V, pipe diameter = D and Poisson's ratio of the ground = ν_S:

$$\Delta D_{flex} = \frac{-8\sigma_v r(1-K)(1-\upsilon_s)^2}{[(3-2\upsilon_s)(1-2\upsilon_s)M_s]} \tag{2}$$

Elastic soil-pipe interaction theory (e.g. Hoeg, 1968 and Moore, 2000) indicates the extent by which pipe deflections and bending moments depend on the stiffness of ground relative to the pipe, normalized using pipe radius (a flexural parameter analogous to the hoop stiffness parameter S_h proposed by McGrath, 1998 and adopted by AASHTO):

$$S_f = \frac{M_s R^3}{E_{pipe} I_{pipe}} \tag{3}$$

Figure 9 shows values of vertical diameter decrease normalized as a proportion of those that develop if the pipe is flexible (ΔD_{flex}), as well as the bending moment as a proportion of the value that develops for a rigid pipe, M_{rigid}. As soil stiffness grows relative to pipe stiffness, the bending moments decrease and the pipe deformations increase. Semi-rigid pipes are those that lie in the region $0.1 < S_f < 1000$, where bending moments are reduced by the earth pressures that result as the soil restrains pipe deformations. Indeed, the diameter change ΔD expressed as a percentage of the flexible pipe limit ΔD_{flex} is a measure of the reduction of bending moments in the structure due to the stiffness of the surrounding soil (applied loads steadily transfer to the ground from the pipe as $\Delta D/\Delta D_{flex}$ increases from 0% to 100%).

The direct way to assess the extent to which the test pipes exhibit semi-rigid response involves use of curvature change measurements to calculate bending moment, and comparison of those bending moments against the bending moments that would occur in rigid pipe. Unfortunately, that moment calculation involves use of $E_{pipe}I_{pipe}$, which is not available. Instead, equation 2 has been employed to calculate the pipe deflections that would occur if the pipe were flexible ΔD_{flex}, using the measured values of soil modulus M_S and lateral earth pressure coefficient K=0.5, and an estimated value of $\nu_S = 0.3$. The measured diameter change ΔD can be normalized using that deflection limit for flexible pipe, placing the test pipe on the interaction curves shown in Figure 9. The bending moment as a proportion of the rigid pipe limit can then be estimated, providing an indication of the benefits gained from semi-rigid soil-pipe interaction.

Table 3 contains selected calculations of ΔD_{flex}, $\Delta D/\Delta D_{flex}$ and $\%M_{rigid}$ for Test pipes 1, 2 and 3. These calculations suggest that:

- The dry test pipe commenced with moments reduced by approximately 4% for this loose granular backfill, and concluded with moments reduced by about 14%

- The lower modulus and thinner manufactured wall thicknesses of the wet test pipes lead to greater benefits from semi-rigid soil-pipe interaction, starting with a moment reduction of approximately 8% and concluding with a reduction of about 26% for Test 3 (at 180kPa overburden pressure), and 38% for Test 2 (at 300kPa overburden pressure)
- During the period of sustained load in Test 3, the estimated moment reductions increased from 26% at 1.5 hours to 31% at 50 hours. The time-dependent response of the fiber reinforced cement material lead to more load being transferred from the pipe to the soil.

These test pipes exhibited semi-rigid pipe behavior, that is these particular structures in this particular backfill obtained significant benefits associated with the soil restraint. Those benefits could be expected to include reductions in bending moments with time, and where a pipe installed dry becomes saturated under field conditions. The loose (dumped) granular backfill used in these soil-box tests represents a relatively low stiffness backfill, and properly engineered DOT type installations should provide higher modulus, McGrath (1998), leading to substantial additional benefits.

Design specifications for FRC pipes currently use rigid pipe design theory based on three-edge bearing loads. Design procedures could continue to employ rigid pipe theory and conservatively neglect the benefits of the semi-rigid pipe behaviour, or changes could be made to pipe design in the future to capitalize on the moment reductions that occur as the pipe deforms and load is transferred to the surrounding ground.

CONCLUSIONS

Soil-pipe interaction tests in a biaxial pipe test cell have been used to examine various aspects of fiber reinforced cement pipe behavior. The tests examined the pipe response in a low stiffness backfill, namely a loose, poorly graded granular material. Three tests were performed to investigate different aspects of the buried pipe behavior: the first test was performed on a dry pipe, and the second and third on wet pipe samples. The first two pipes were tested to maximum overburden pressures of 240kPa and 300kPa respectively. The third pipe test featured a period monitoring the pipe response under sustained overburden pressure of 180kPa, to examine the nature of the ongoing, time-dependent pipe deformations.

Pipe deformations increased almost linearly with overburden pressure, but remained below 1% for overburden pressures up to 180kPa. Deformations for the dry sample were about half those for the wet samples, indicating that the modulus of the dry FRC pipe was about 15% greater than that for the wet pipes (allowing for the higher manufactured wall thickness of the dry pipe sample). The measured values of circumferential strain on the inner and outer surfaces were used to assess hoop strain and changes in pipe curvature. These indicate that for the almost uniform burial conditions used in the laboratory, the bending moments at Crown and Invert are almost equal and opposite to those at the Springlines. Changes in curvature measured at the haunches were an order of magnitude lower, as would be expected for a pipe of relatively high stiffness.

Measurements of the flexural rigidity of the test pipes are not available, making a direct calculation of the bending moments as a proportion of those moments that develop in an equivalent rigid pipe impossible. Therefore the benefits of semi-rigid soil-pipe interaction enjoyed by the test pipes were estimated using pipe deformations. Deformations for a flexible pipe buried in this configuration were calculated using elastic soil-pipe interaction theory and the measured values of constrained soil modulus, and these flexible pipe deflections were used to normalize the pipe diameter changes measured for the test pipes. Since the flexible pipe limit represents the situation where all the non-uniform ground stresses are transferred to the surrounding soil, the normalized deflection indicates the level of moment reduction resulting from soil restraint. These estimates indicate that the test pipes enjoyed moment reductions of between 4% and 8% for the low soil modulus of 2.8 MPa that the loose backfill provided at these low overburden pressures. Moment reductions increased to 14% and 26% as the test pipes were loading to levels closer to their design limits. The additional ground support enjoyed by the wet pipe samples arose from the reduced wall thickness of those samples and the lower modulus of saturated FRC.

Time dependent changes in the modulus of the saturated FRC lead to a 12% increase in pipe deformations by the end of the test at 50 hours. Measured increases in pipe curvature were about half that value. These additional deflections over time observed during the sustained-load portion of test 3 are estimated to have enhanced moment reductions to 30% by the end of the 50 hour period.

Work is ongoing to undertake analysis of the soil-pipe interaction using elastic theory and nonlinear finite element analysis. Those studies will include assessment of the effect of pipe wall thickness, wet versus dry pipe response and the effect of time on the bending moments that develop under field conditions. All results reported here are pertinent to the specific pipe samples and burial conditions employed in these tests.

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Table 1. Circumferential strains (in microstrain) on the inner and outer surface of Test pipes 1 to 3; two test sections (A and B); discarded values shown with ~~strikethrough~~.

	Springlines				Crown		Invert		Haunch	
Test 1 at 1.7 hours and 240kPa	A right	A left	B right	B left	A	B	A	B	A	B
Inner	-988	-1070	-1177	-1085	716	898	975	722	-41	-123
Outer	~~541~~[2]	823	805	827	~~0~~	-1230	-857	~~0~~	-140	34
Test 2 at 2 hours and 300kPa	A right	A left	B right	B left	A	B	A	B	A	B
Inner	~~0~~	-2345	-2797	-2271	2226	4548[3]	~~0~~	~~575~~[2]	~~0~~	-339
Outer	1750	~~0~~	~~530~~[2]	~~849~~[2]	~~0~~	-2384	-2368	-2432	83	46
Test 3 at 2 hours and 180kPa	A right	A left	B right	B left	A	B	A	B	A	B
Inner	-1391	-1352	~~0~~	~~0~~	~~0~~	~~0~~	~~171~~	~~3660~~[2]	-133	-223
Outer	~~0~~	776	~~277~~[1]	0	-1170	-1168	~~0~~	~~0~~	~~0~~	13
Test 3 at 50 hours and 180kPa	A right	A left	B right	B left	A	B	A	B	A	B
Inner	-1518	~~-697~~[1]	~~0~~	~~0~~	~~0~~	~~0~~	~~-535~~[1]	~~5027~~[2]	~~879~~[2]	~~0~~
Outer	~~0~~	730	~~359~~[1]	0	-1298	-1263	~~0~~	~~0~~	~~0~~	~~-783~~[2]

Note: [1]: value discarded because of discrepancy relevant to other measures.

[2]: value discarded due to lack of continuity in total strain record.

[3]. value discarded since it implies the presence of a large tensile hoop force.

Table 2. Representative (i.e. averaged) surface strains in microstrain; hoop strains in microstrain; change in curvature in mm^{-1}; Test pipes 1 and 3.

Test 1 at 240kPa, 1.7 hours			
	Springlines	Crown/Invert	Haunch
Inner fiber	-1080	827	-82
Outer fiber	818	-1044	-53
ε_h	-131	-108	-67
κ mm^{-1}	-6.6E-05	6.5E-05	-1.0E-06
Test 2 at 300kPa, 2 hours			
	Springlines	Crown/Invert	Haunch
Inner fiber	-2471	2226	-339
Outer fiber	1750	-2395	65
ε_h	-361	-84	-137
κ mm^{-1}	-1.5E-04	1.6E-04	-1.4E-05
Test 3 at 180kPa, 2 hours			
	Springlines	Crown/Invert	Haunch
Inner fiber	-1372	*776*[1]	-178
Outer fiber	776	-1169	13
ε_h	-298	-197	-83
κ mm^{-1}	-7.5E-05	*6.8E-05*[1]	-6.7E-06
Test 3 at 180kPa, 50 hours			
	Springlines	Crown/Invert	Haunch
Inner fiber	-1518	*730*	NA
Outer fiber	730	-1281	NA
ε_h	-394	-275	NA
κ mm^{-1}	-7.9E-05	*7.0E-05*[1]	NA

Note [1]: no reasonable measurements were obtained during Test 3 for inner fiber stresses at the crown and invert; the outer fiber stress at the springline was therefore used in the calculation of curvature.

Table 3. Analysis of displacement relative to flexible pipe limit, and moment relative to rigid pipe limit; calculations based on Moore, 2000; deflections taken eight minutes after the start of selected pressure levels.

Test	σ_V kPa	ΔD_{test} mm	M_S kPa	ΔD_{flex} mm	$\%\Delta D_{flex}$	$\%M_{rigid}$
Test 1 Dry pipe	40	-0.1	2759	-2.6	4%	96%
	100	-0.7	3979	-7.4	9%	91%
	160	-1.2	5572	-10.3	11%	89%
	240	-1.9	6905	-13.2	14%	86%
Test 2 Wet pipe	80	-1.1	3674	-9.1	12%	88%
	140	-2.4	4865	-12.0	20%	80%
	200	-3.8	5863	-14.2	27%	73%
	300	-6.7	7195	-17.4	38%	62%
Test 3 Wet pipe	20	-0.2	2674	-3.2	8%	92%
	80	-1.1	4689	-7.2	15%	85%
	140	-2.1	6155	-9.5	22%	78%
	180	-2.9	6986	-10.8	26%	74%

Table 4. Analysis of time dependent increase in displacement and estimated decrease in proportion of rigid pipe moment; sustained load test on wet pipe (calculations based on Moore, 2000).

time hrs	ΔD_{test} mm	M_S kPa	ΔD_{flex} mm	$\%\Delta D_{flex}$	$\%M_{rigid}$
1.5	-2.9	6986	-10.8	26%	74%
2	-3.0	6986	-10.8	27%	73%
4	-3.1	6986	-10.8	28%	72%
10	-3.2	6986	-10.8	30%	70%
25	-3.4	6986	-10.8	31%	69%
50	-3.4	6986	-10.8	31%	69%

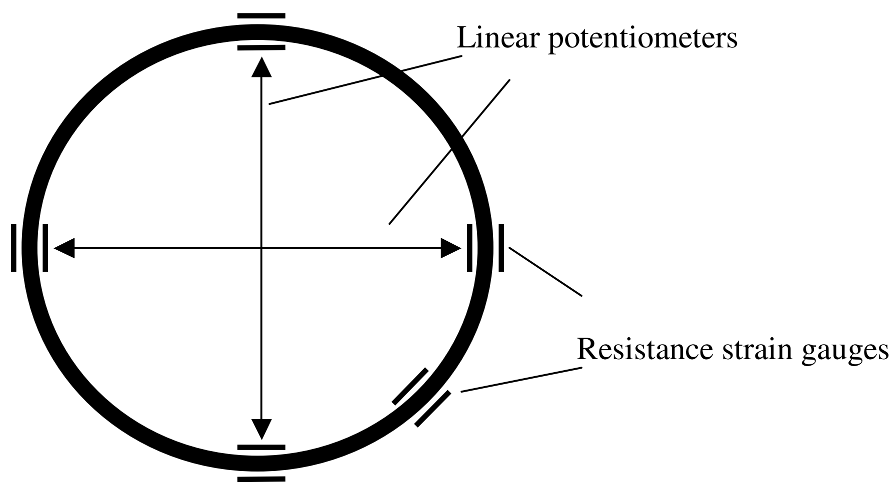

Figure 1. Pipe instrumentation (identical at both sections A and B).

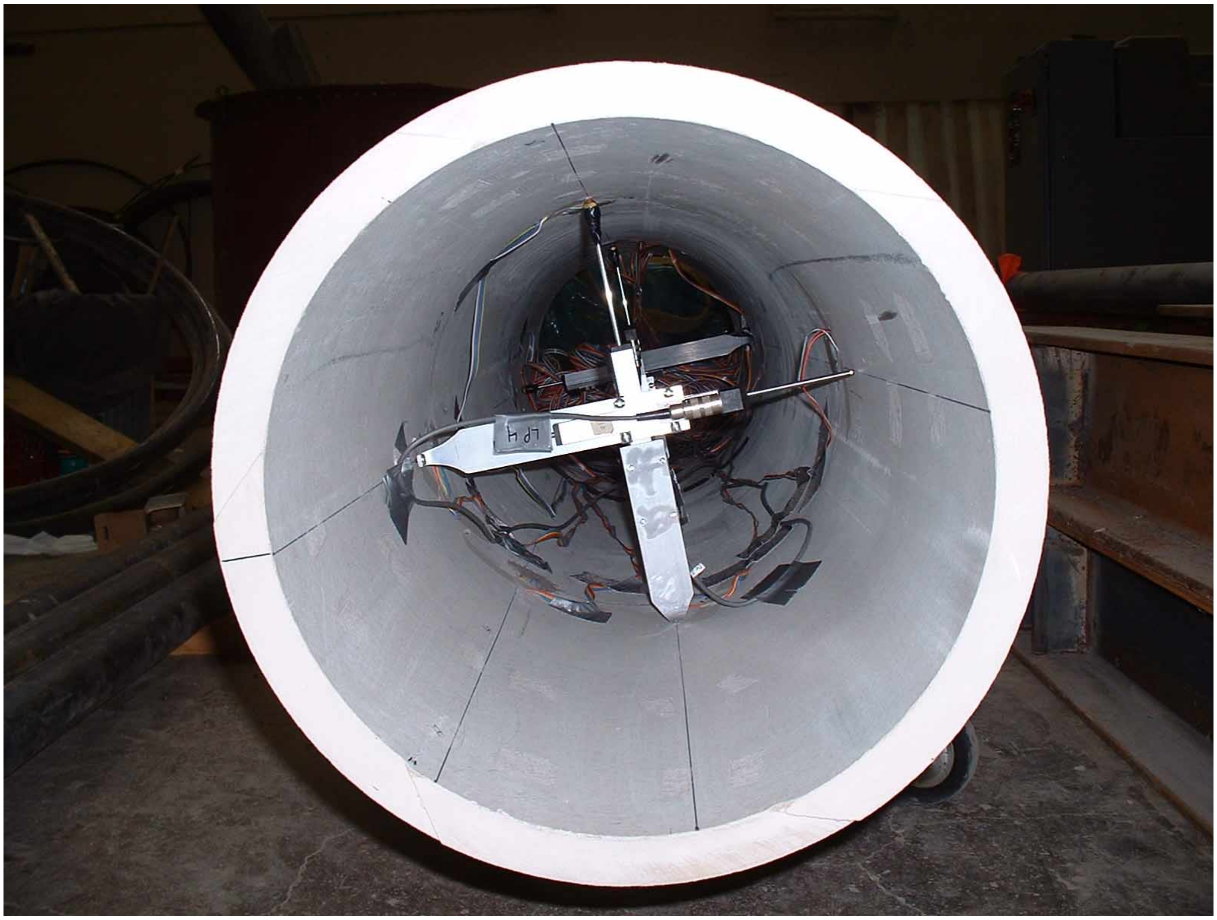

Figure 2. End of dry pipe specimen used in Test 1 showing linear potentiometers and resistance strain gauges on inner surface.

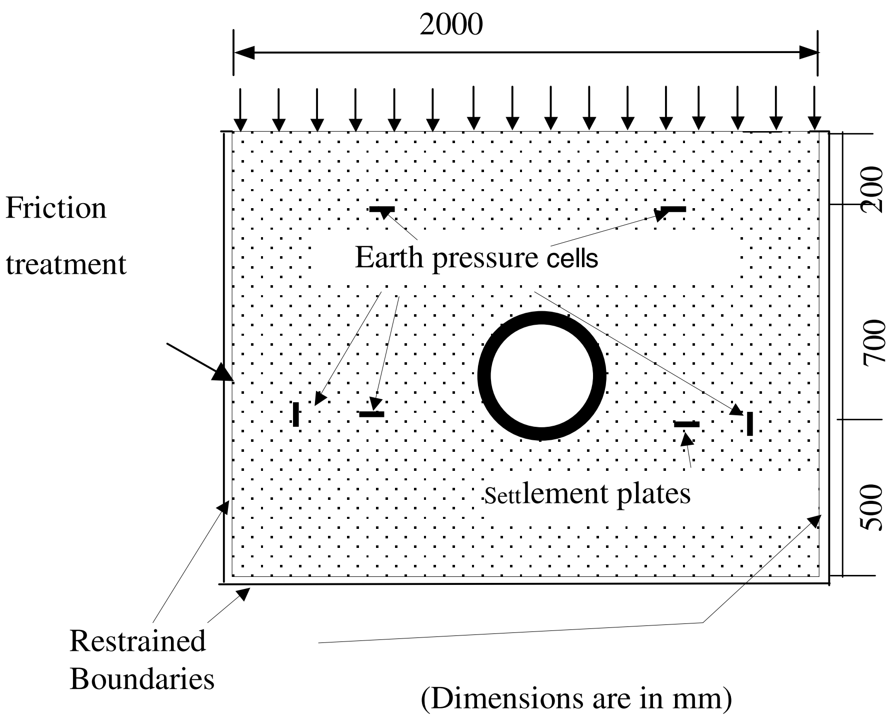

Figure 3. Location of the pipe and the soil instrumentation in the test cell

Figure 4. Dry pipe specimen being buried in the test cell; image shows geosynthetic used to protect sidewall friction treatment.

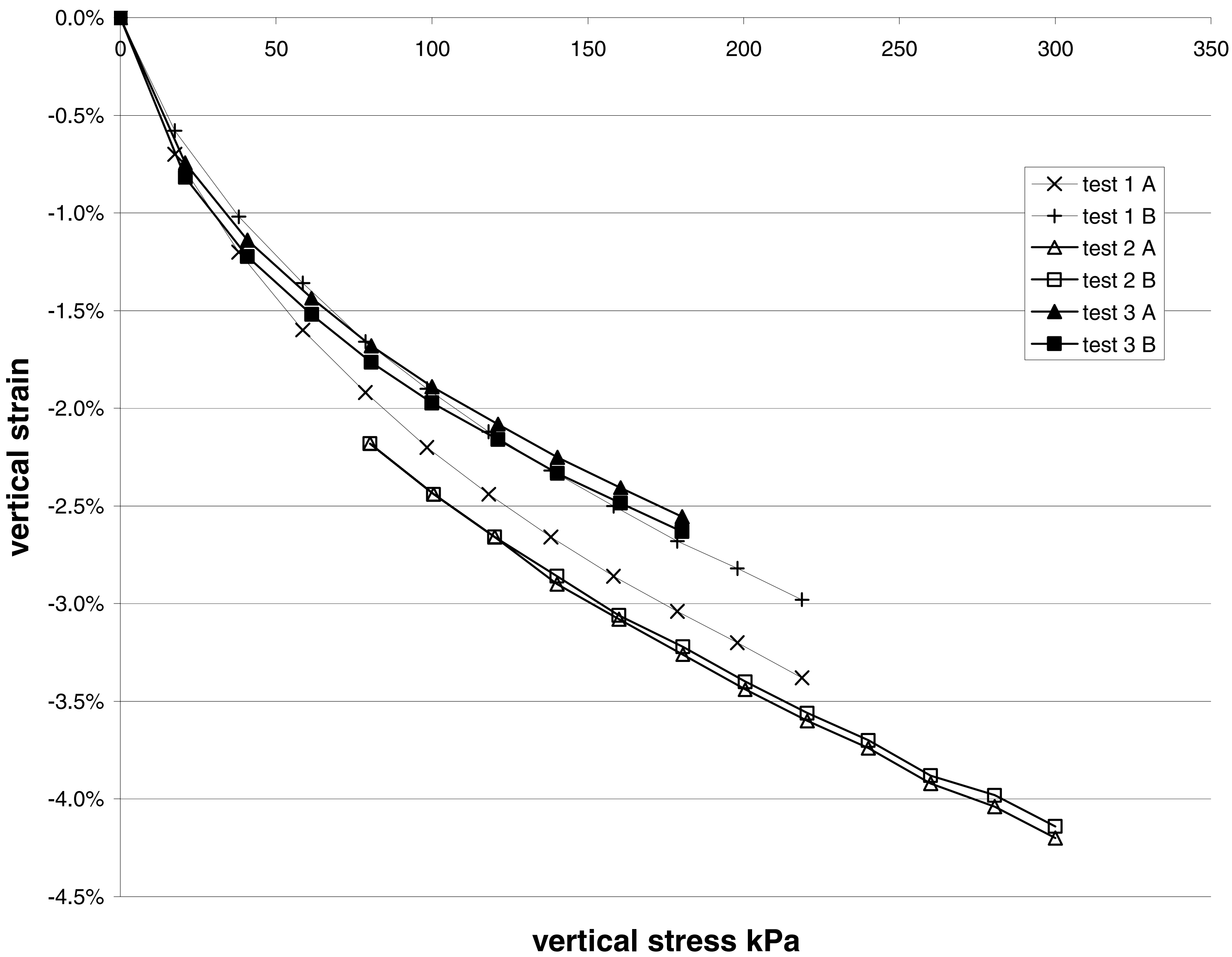

Figure 5. Vertical stress versus vertical strain in the soil column near the side of the cell; tests 1 to 3, at sections A and B.

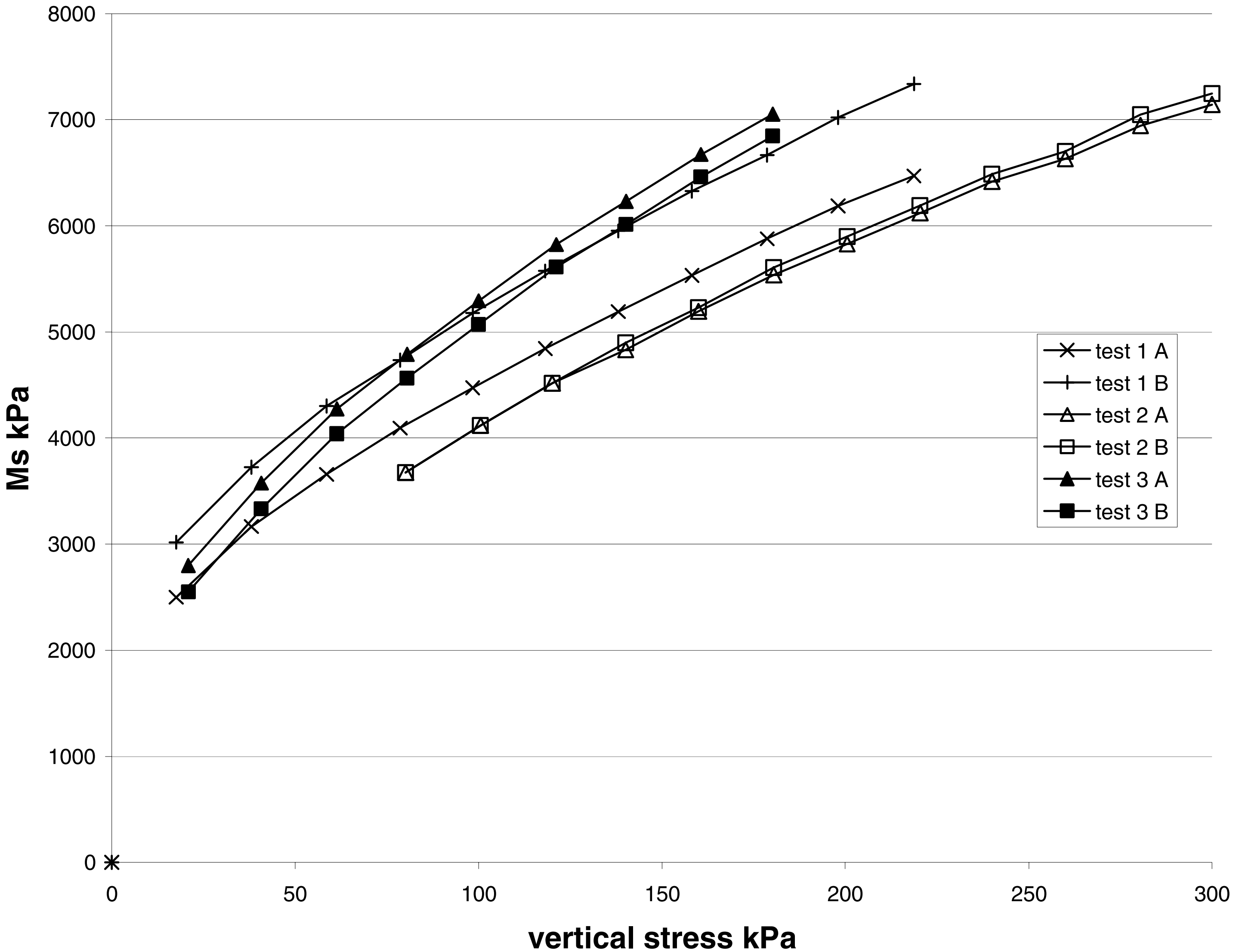

Figure 6. Constrained (one dimensional vertical) modulus for soil column at sides of cell; tests 1 to 3, Sections A and B.

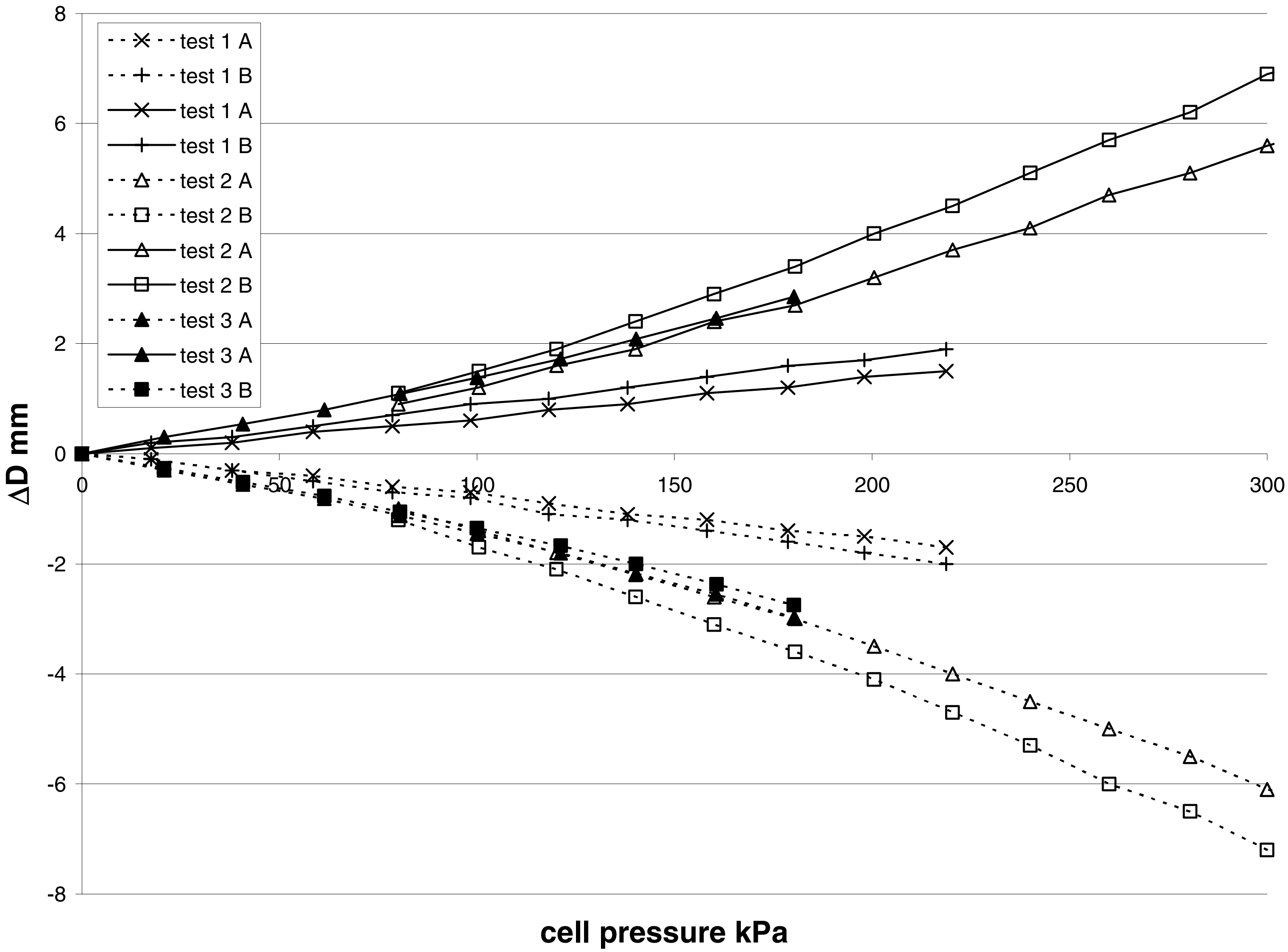

Figure 7. Change in pipe diameter; tests 1 to 3 at sections A and B (measurements at the end of each 8 minute loading period).

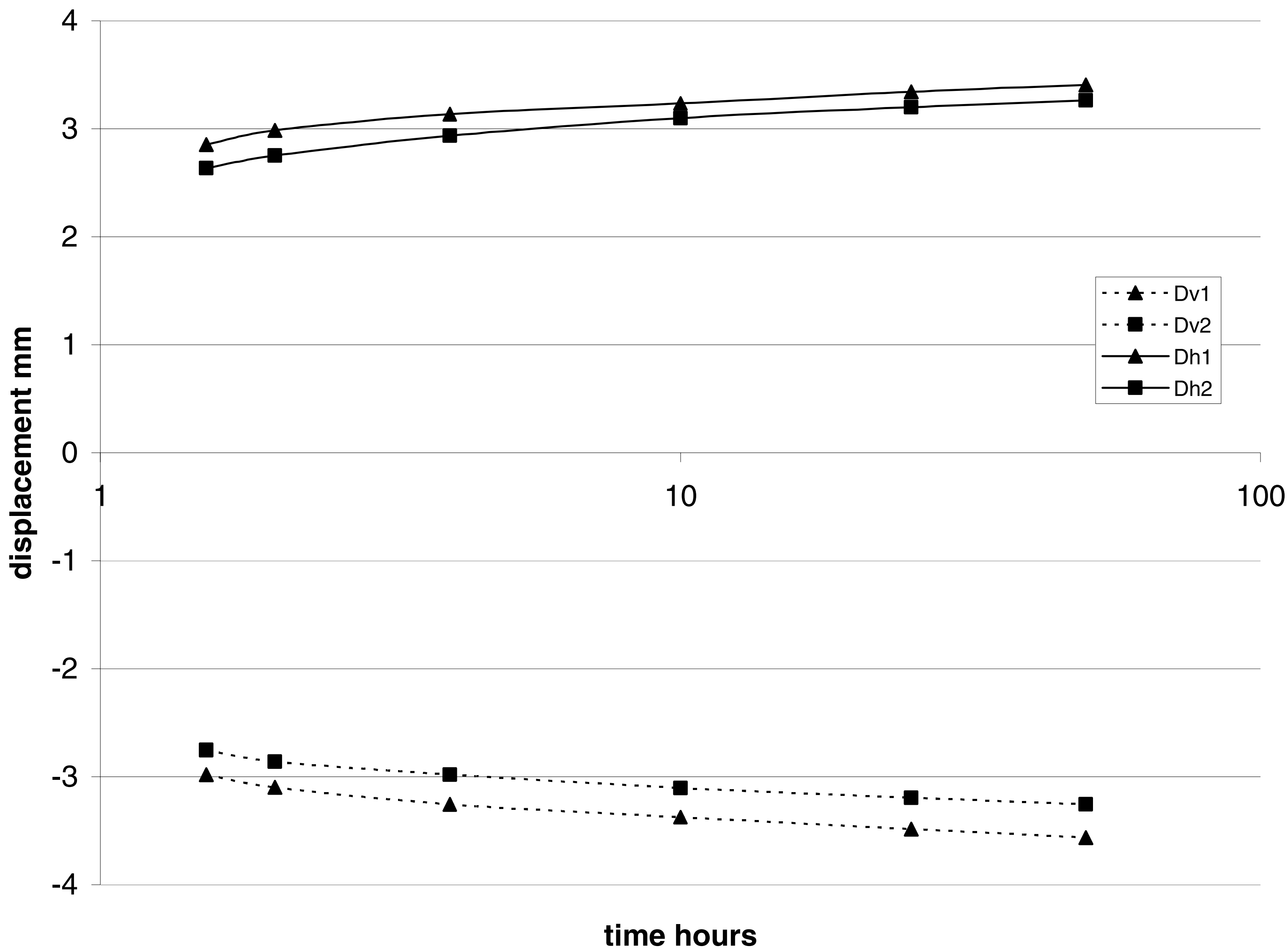

Figure 8. Continued deflection of extended test on wet pipe (Test 3).

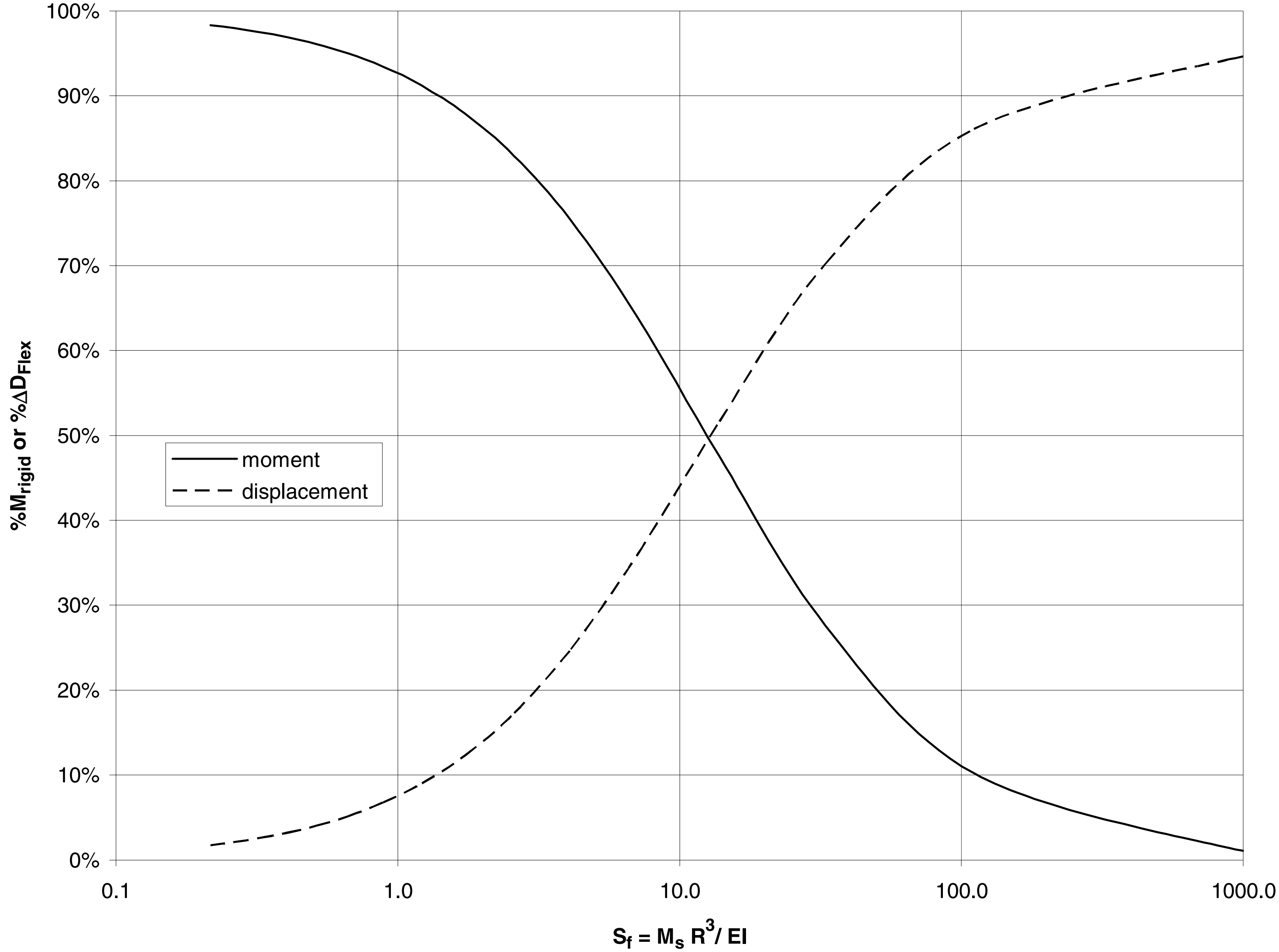

Figure 9. Relative moment and relative deflection for semi-rigid pipe (theoretical calculations based on Moore, 2000).